Chapter 9   Bearings & Expansion Joints   Contents

9.1 Expansion Joints

9.1.1 General Considerations

All bridges must accommodate, in some manner, environmentally and self-imposed phenomena that tend to make structures move in various ways. These movements come from several primary sources: thermal variations, concrete shrinkage, creep effects from prestressing, and elastic post-tensioning shortening. With the exception of elastic post-tensioning shortening, which generally occurs before expansion devices are installed, movements from these primary phenomena are explicitly calculated for expansion joint selection and design. Other movement inducing phenomena include live loading (vertical and horizontal braking), wind, seismic events, and foundation settlement. Movements associated with these phenomena are generally either not calculated or not included in total movement calculations for purposes of determining expansion joint movement capacity.

With respect to seismic movements, it is assumed that some expansion joint damage may occur, that this damage is tolerable, and that it will be subsequently repaired. In cases where seismic isolation bearings are used, the expansion joints must accommodate seismic movements in order to allow the isolation bearings to function properly.

Expansion joints must accommodate cyclic and long-term structure movements in such a way as to minimize imposition of secondary stresses in the structure. Expansion joint devices must prevent water, salt, and debris infiltration to substructure elements below. Additionally, an expansion joint device must provide a relatively smooth riding surface over a long service life.

Expansion joint devices are highly susceptible to vehicular impact that results as a consequence of their inherent discontinuity. Additionally, expansion joints have often been relegated a lower level of importance by both designers and contractors. Many of the maintenance problems associated with in-service bridges relate to expansion joints.

One solution to potential maintenance problems associated with expansion joints is to use construction procedures that eliminate the joints from the bridge deck. The two most commonly used methods are called integral and semi-integral construction. These two terms are sometimes collectively referred to as jointless bridge construction. In integral construction, concrete end diaphragms are cast monolithically with both the bridge deck and supporting pile substructure. In order to minimize secondary stresses induced in the superstructure, steel piles are generally used in their weak axis orientation relative to the direction of bridge movement. In semi-integral construction, concrete end diaphragms are cast monolithically with the bridge deck. Supporting girders rest on elastomeric bearings within an L-type abutment. Longer semi-integral bridges generally have reinforced concrete approach slabs at their ends. Approach slab anchors, in conjunction with a compression seal device, connect the monolithic end diaphragm to the approach slab. Longitudinal movements are accommodated by diaphragm movement relative to the approach slab, but at the same time resisted by soil passive pressure against the end diaphragm.

Obviously, bridges cannot be built incrementally longer without eventually requiring expansion joint devices. The incidence of approach pavement distress problems increases markedly with increased movement that must be accommodated by the end diaphragm pressing against the backfill. Approach pavement distress includes pavement and backfill settlement and broken approach slab anchors.

Washington State Department of Transportation (WSDOT) has implemented jointless bridge design by using semi-integral construction. Office policy for concrete and steel bridge design is as follows:

A. Concrete Bridges: Semi-integral design is used for prestressed concrete girder bridges under 450 feet long and for post-tensioned spliced concrete girder and cast-in-place post-tensioned concrete box girder bridges under 400 feet long. Use L-type abutments with expansion joints at the bridge ends where bridge length exceeds these values. In situations where bridge skew angles exceed 30 degrees, consult the Bearing and Expansion Joint Specialist and the Bridge Design Engineer for recommendations and approval.

B. Steel Bridges: Use L-type abutments with expansion joints at the ends for multiple-span bridges. *Semi-integral* construction may be used in lieu of expansion joints for single span bridges under 300 feet with the approval of the Bridge Design Engineer. In situations where the bridge skew exceeds 30 degrees, consult the Bearing and Expansion Joint Specialist and the Bridge Design Engineer for recommendations and approval.

In all instances, the use of intermediate expansion joints should be avoided wherever possible. The following table provides guidance regarding maximum bridge superstructure length beyond which the use of either intermediate expansion joints or modular expansion joints at the ends is required.

Superstructure Type	Maximum Length (western WA)		Maximum Length (eastern WA)	
	Semi-Integral	L-Abutment	Semi-Integral	L-Abutment
Concrete Superstructure				
Prestressed girder*	450 ft.	900 ft.	450 ft.	900 ft.
P.T. spliced girder**	400 ft.	700 ft.***	400 ft.	700 ft.***
C.I.P. - P.T. box girder	400 ft.	700 ft. ***	400 ft.	700 ft.***
Steel Superstructure				
Plate girder Box girder	300 ft.	1,000 ft.	300 ft.	800 ft.

*   Based upon 0.16 in. creep shortening per 100 ft. of superstructure length and 0.12 in. shrinkage shortening per 100 ft. of superstructure length

**   Based upon 0.31 in. creep shortening per 100 ft. of superstructure length and 0.19 in. shrinkage shortening per 100 ft. of superstructure length

***   Can be increased to 800 ft. if the joint opening at 64° F at time of construction is specified in the expansion joint table to be less than the minimum installation width of 1½ in. This condition is acceptable if the gland is already installed when steel shapes are placed in the blockout. Otherwise (for example, staged construction) the gland would need to be installed at temperature less than 45° F.

Because the movement restriction imposed by a bearing must be compatible with the movements allowed by the adjacent expansion joint, expansion joints and bearings must be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

A plethora of manufactured devices exists to accommodate a wide range of expansion joint total movements. Expansion joints can be broadly classified into three categories based upon their total movement range as follows:

Small Movement Joints   Total Movement Range < 1¾ in.

Medium Movement Joints   1¾ in. < Total Movement Range < 5 in.

Large Movement Joints   Total Movement Range > 5 in.

9.1.2   General Design Criteria

Expansion joints must be sized to accommodate the movements of several primary phenomena imposed upon the bridge following installation of its expansion joint devices. Concrete shrinkage, thermal variation, and long-term creep are the three most common primary sources of movement. Calculation of the movements associated with each of these phenomena must include the effects of superstructure type, tributary length, fixity condition between superstructure and substructure, and pier flexibilities.

A. Shrinkage Effects

Accurate calculation of shrinkage as a function of time requires that average ambient humidity, volume-to-surface ratios, and curing methods be taken in consideration as summarized in LRFD Article 5.4.2.3.3. Because expansion joint devices are generally installed in their respective blockouts at least 30 to 60 days following concrete deck placement, they must accommodate only the shrinkage that occurs from that time onward. For most situations, that shrinkage strain can be assumed to be 0.0002 for normal weight concrete in an unrestrained condition. This value must be corrected for restraint conditions imposed by various superstructure types.

$$\Delta_{shrink} = \beta \cdot \mu \cdot L_{trib}$$

where

L_{trib} = tributary length of the structure subject to shrinkage

β = ultimate shrinkage strain after expansion joint installation; estimated as 0.0002 in lieu of more refined calculations

μ = restraint factor accounting for the restraining effect imposed by superstructure elements installed before the concrete slab is cast

= 0.0 for steel girders, 0.5 for precast prestressed concrete girders, 0.8 for concrete box girders and T-beams, 1.0 for concrete flat slabs

B. Thermal Effects

Bridges are subject to all modes of heat transfer: radiation, convection, and conduction. Each mode affects the thermal gradients generated in a bridge superstructure differently. Climatic influences vary geographically resulting in different seasonal and diurnal average temperature variations. Additionally, different types of construction have different thermal "inertia" properties. For example, a massive concrete box girder bridge will be much slower to respond to an imposed thermal stimulus, particularly a diurnal variation, than would a steel plate girder bridge composed of many relatively thin steel elements.

Variation in the superstructure average temperature produces elongation or shortening. Therefore, thermal movement range is calculated using the maximum and minimum anticipated bridge superstructure average temperatures anticipated during the structure's lifetime. The considerations in the preceding paragraph have led to the following maximum and minimum anticipated bridge superstructure average temperature guidelines for design in Washington State:

Concrete Bridges:	0° F to 100° F
Steel Bridges (eastern Washington)	-30° F to 120° F
Steel Bridges (western Washington)	0° F to 120° F

Total thermal movement range is then calculated as:

$$\Delta_{temp} = \alpha \cdot L_{trib} \cdot \delta T$$

where

L_{trib} = tributary length of the structure subject to thermal variation

α = coefficient of thermal expansion; 0.000006 in./in./°F for concrete and 0.0000065 in./in./°F for steel

δT bridge superstructure average temperature range as a function of bridge type and location

In accordance with WSDOT Standard Specifications, contract drawings state dimensions at a normal temperature of 64° F unless specifically noted otherwise. Construction and fabrication activities at average temperatures other than 64° F require the Contractor or fabricator to adjust lengths of structural elements and concrete forms accordingly.

Some expansion joint devices are installed in pre-formed concrete blockouts some time after the completion of the bridge deck. The expansion joint device must be cast into its respective blockout with a gap setting corresponding to the ambient superstructure average temperature at the time the blockouts are filled with concrete. In order to accomplish this, expansion device gap settings must be specified on the contract drawings as a function of superstructure ambient average temperature. Generally, these settings are specified for temperatures of 40° F, 64° F, and 80° F.

9.1.3 Small Movement Range Joints

Elastomeric compression seals, poured sealants, asphaltic plugs, pre-formed closed cell foam, epoxy-bonded elastomeric glands, steel sliding plates, and bolt-down elastomeric panels have all been used in the past for accommodating small movement ranges. The current policy is to use compression seals and rapid-cure silicone sealants almost exclusively.

A. Compression Seals

Compression seals are continuous manufactured elastomeric elements, typically with extruded internal web systems, installed within an expansion joint gap to effectively seal the joint against water and debris infiltration. Compression seals are held in place by mobilizing friction against adjacent vertical joint faces. Design philosophy requires that they be sized and installed to always be in a state of compression.

Compression seals can be installed against smooth vertical concrete faces or against steel armoring. When installed against concrete, special concrete nosing material having enhanced impact resistance is typically used. Polymer concrete, polyester concrete, and elastomeric concrete have been used with varying degrees of successful performance. Consult the Bearing and Expansion Joint Specialist for current policy.
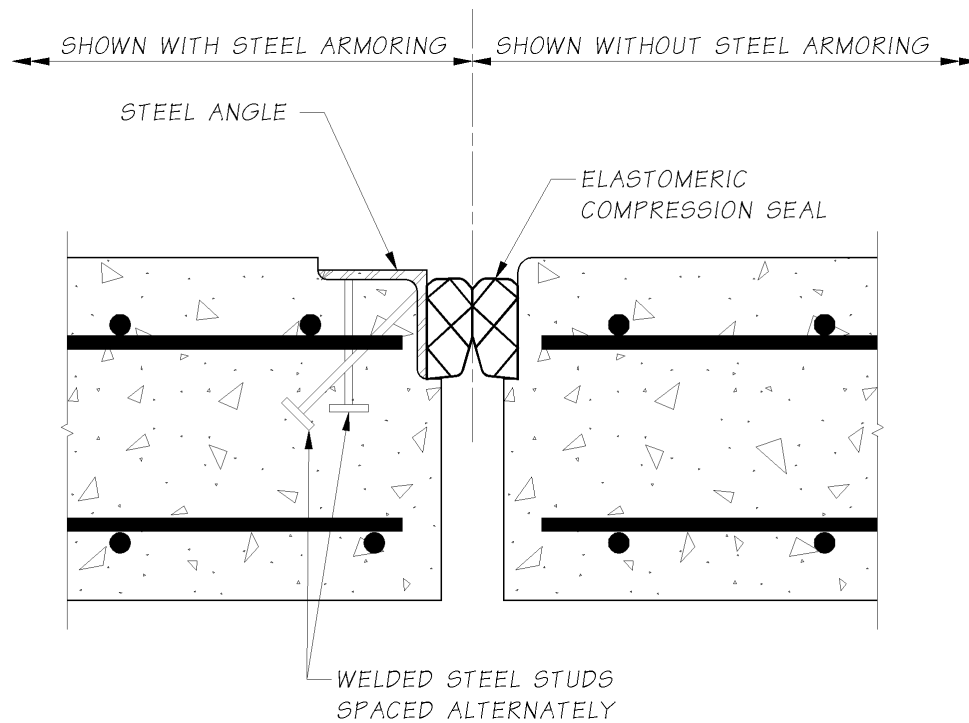
Compression Seal Joint
Figure 9.1.3-1

In design calculations, the minimum and maximum compressed widths of the seal are generally set at 40 percent and 85 percent of the uncompressed width. These measurements are perpendicular to the joint axis. It is generally assumed that the compressed seal width at the normal construction temperature of 64° F is 60 percent of its uncompressed width. For skewed joints, bridge deck movement must be separated into components perpendicular to and parallel to the joint axis. Shear displacement of the compression seal should be limited to a specified percentage of its uncompressed width, usually set at about 22 percent. Additionally, the expansion gap width should be set so that the compression seal can be replaced over a reasonably wide range of construction temperatures. Manufacturers' catalogues generally specify the minimum expansion gap widths into which specific size compression seals can be installed. The expansion gap width should be specified on the contract drawings as a function of the superstructure average temperature.

Compression seal movement design relationships can be expressed as:

$$\Delta_{\text{temp-normal}} = \Delta_{\text{temp}} \cdot \cos \theta \; [\text{thermal movement normal to joint}]$$
$$\Delta_{\text{temp-parallel}} = \Delta_{\text{temp}} \cdot \sin \theta \; [\text{thermal movement parallel to joint}]$$
$$\Delta_{\text{shrink-normal}} = \Delta_{\text{shrink}} \cdot \cos \theta \; [\text{shrinkage movement normal to joint}]$$
$$\Delta_{\text{shrink-parallel}} = \Delta_{\text{shrink}} \cdot \sin \theta \; [\text{shrinkage movement parallel to joint}]$$
$$W_{\text{min}} = W_{\text{install}} - [(T_{\text{max}}\text{-}T_{\text{install}})/(T_{\text{max}}\text{-}T_{\text{min}})] \cdot \Delta_{\text{temp-normal}} > 0.40 \cdot W$$
$$W_{\text{max}} = W_{\text{install}} + [(T_{\text{install}}\text{-}T_{\text{min}})/(T_{\text{max}}\text{-}T_{\text{min}})] \cdot \Delta_{\text{temp-normal}} + \Delta_{\text{shrink-normal}} < 0.85 \cdot W$$

where θ = skew angle of the expansion joint, measured with respect to a line perpendicular to the bridge longitudinal axis

W   =   uncompressed width of the compression seal

$W_{install}$ =   expansion gap width at installation

$T_{install}$ =   superstructure temperature at installation

W_{min} =   minimum expansion gap width

W_{max} =   maximum expansion gap width

T_{min} =   minimum superstructure average temperature

T_{max} =   maximum superstructure average temperature

Algebraic manipulation yields:

$W > (\Delta_{temp\text{-}normal} + \Delta_{shrink\text{-}normal})/0.45$

$W > (\Delta_{temp\text{-}parallel} + \Delta_{shrink\text{-}parallel})/0.22$

Now, assuming $W_{install} = 0.6 \cdot W$,

$W_{max} = 0.6 \cdot W + [(T_{install} - T_{min})/(T_{max} - T_{min})] \cdot \Delta_{temp\text{-}normal} + \Delta_{shrink\text{-}normal} < 0.85 \cdot W$

Rearranging yields:

$W > 4 \cdot [(T_{install} - T_{min})/(T_{max} - T_{min}) \cdot \Delta_{temp\text{-}normal} + \Delta_{shrink\text{-}normal}]$

Design Example:

Given: A reinforced concrete box girder bridge has a total length of 200 ft. A compression seal expansion joint at each abutment will accommodate half of the total bridge movement. The abutments and expansion joints are skewed 15°. Bridge superstructure average temperatures are expected to range between 0° F and 100° F.

Find: Required compression seal size and construction gap widths at 40° F, 64° F, and 80° F.

Solution:

Step 1:   Calculate temperature and shrinkage movement.

Temperature: $\Delta_{temp} = \frac{1}{2}(.000006)(100°F)(200')(12''/')$ $= 0.72''$
Shrinkage: $\Delta_{shrink} = \frac{1}{2}(.0002)(0.8)(200')(12''/')$ $= \underline{0.19''}$
Total deck movement at the joint: $= 0.91''$

$\Delta_{temp\text{-}normal} + \Delta_{shrink\text{-}normal} = (0.91'')(\cos 15°) = 0.88''$

$\Delta_{temp\text{-}parallel} + \Delta_{shrink\text{-}parallel} = (0.91'')(\sin 15°) = 0.24''$

Step 2:   Determine compression seal width required.

$W > 0.88''/0.45 = 1.96''$

$W > 0.24''/0.22 = 1.07''$

$W > 4[(64°F - 0°F)/(100°F - 0°F) \cdot (0.72'') + 0.19''] (\cos 15°) = 2.51''$
$\rightarrow$Use a 3'' compression seal

Step 3:   Evaluate construction gap widths for various temperatures for a 3 in. compression seal.

Construction width at 64° F = 0.6(3'') = 1.80''

Construction width at 40° F = 1.80'' + [(64° - 40°)/(100° + 0°)]·(0.72'')·(cos 15°) = 1.97''

Construction width at 80° F = 1.80'' − [(80° - 64°)/(100° + 0°)]·(0.72'')·(cos 15°) = 1.69''

Conclusion: Use a 3 in. compression seal. Construction gap widths for installation at temperatures of 40° F, 64° F, and 80° F are 2 in., 1-13/16 in., and 1-11/16 in. respectively.
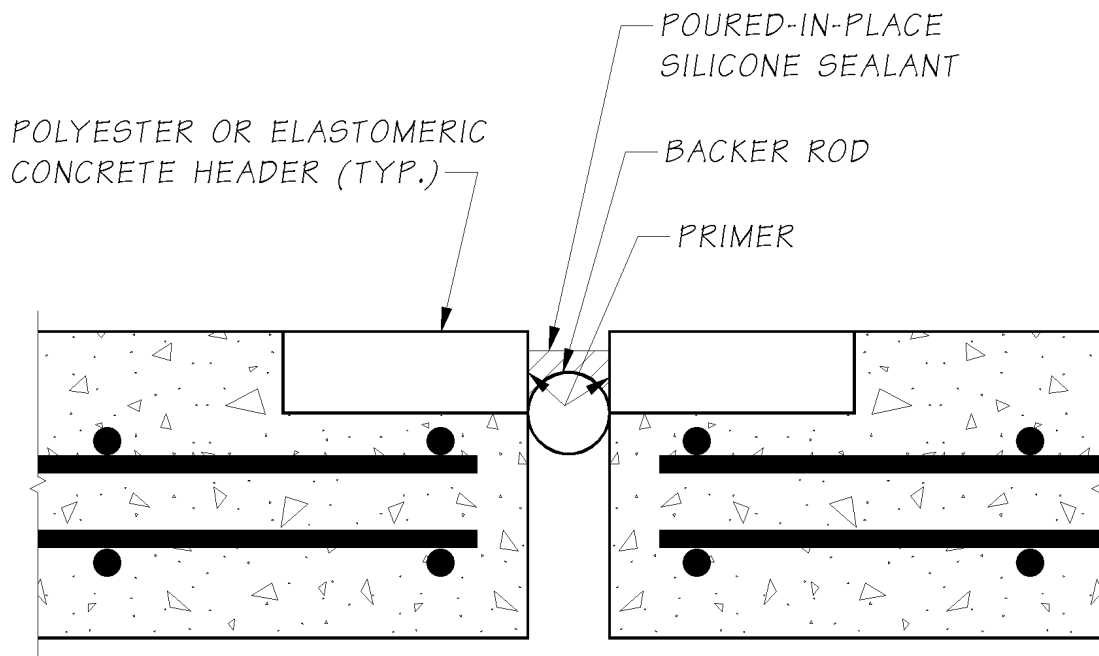
B. Rapid-Cure Silicone Sealants

Durable low-modulus poured sealants provide watertight expansion joint seals in both
new construction and rehabilitation projects. Most silicone sealants possess good elastic
performance over a wide range of temperatures while demonstrating high levels of resistance
to ultraviolet and ozone degradation. Other desirable properties include self-leveling and self-
bonding characteristics.

Rapid-cure silicone sealants are particularly good candidates for rehabilitation in situations
where significant traffic disruption consequential to extended traffic lane closure is unacceptable.
Additionally, unlike compression seals, rapid-cure silicone sealants do not require straight,
parallel substrate surfaces in order to create a watertight seal.

Rapid-cure silicone sealants can be installed against either concrete or steel. It is extremely
critical that concrete or steel substrates be thoroughly cleaned before the sealant is installed. Some
manufacturers require application of specific primers onto substrate surfaces prior to sealant
installation in order to enhance bonding. Consult the Bearing and Expansion Joint Specialist
for specifics.

Rapid-cure Silicone Sealants Joint
Figure 9.1.3-2

Rapid-cure silicone sealants should be designed based upon the manufacturer's recommendations.
Maximum and minimum working widths of the poured sealant joint are generally recommended
as a percentage of the sealant width at installation. Depending upon the manufacturer, these
joints can accommodate tensile movements of up to 100 percent and compressive movements
of up to 50 percent of the sealant width at installation. A minimum recess is typically required
between the top of the roadway surface and the top of the sealant surface. This recess is critical
in assuring that tires will not contact the top surface of the sealant and initiate its debonding from
substrate material.

Design Example:

Given: An existing 25-year old 160 ft. long single span prestressed concrete girder bridge is scheduled for a concrete overlay. The existing compression seals at each non-skewed abutment are in poor condition, although the existing concrete edges on each side of each expansion joint are in relatively good condition. The expansion gaps at these abutments are 1 in. wide at a normal temperature of 64° F. Assume that each expansion joint will accommodate half of the total bridge movement. Bridge superstructure average temperatures are expected to range between 0° F and 100° F.

Find: Determine the feasibility of reusing the existing 1 in. expansion gaps for a rapid cure silicone sealant system retrofit. Assume that the sealant will be installed at an average superstructure temperature between 40° F and 80° F. Manufacturer's recommendations state that Sealant A can accommodate 100 percent tension and 50 percent compression and that Sealant B can accommodate 50 percent tension and 50 percent compression.

Solution:

Step 1: Calculate future temperature, shrinkage, and creep movements.

Temperature: Δ_{temp} = ½ (.000006)(100°F)(160′)(12″/′) = 0.58″

Shrinkage: Δ_{shrink} = 0 (Essentially all shrinkage has already occurred.)

Creep: Δ_{creep} = 0 (Essentially all creep has already occurred.)

Step 2: Calculate existing expansion gap widths at average superstructure temperatures of 40°F and 80°F. These are estimated extreme sealant installation temperatures.

G_{40F} = 1.00″ + [(64° F-40° F)/(100° F-0°F)]·(.58″) = 1.14″

G_{80F} = 1.00″ – [(80° F-64° F)/(100° F-0° F)]·(.58″) = 0.91″

Step 3: Check sealant capacity if installed at 40°F.

Closing movement = [(100° F – 40° F)/(100°F – 0° F)](.58″) = 0.35″
0.35″/1.14″ = 0.31 < 0.50 Sealants A and B

Opening movement = [(40° F – 0° F)/(100° F – 0° F)](.58″) = 0.23″
0.23″/1.14″ = 0.20 < 1.00 Sealant A< 0.50 Sealant B

Step 4: Check sealant capacity if installed at 80° F.

Closing movement = = [(100° F – 80° F)/(100° F – 0° F)](.58″) = 0.12″
0.12″/0.91″ = 0.13 < 0.50 Sealants A and B

Opening movement = [(80° F – 0° F)/(100° F – 0° F)](.58″) = 0.46″
0.46″/0.91″ = 0.50 < 1.00 Sealant A
= 0.50 Sealant B

Conclusion: The existing 1 in. expansion gap is acceptable for installation of a rapid cure silicone sealant system. Note that Sealant B would reach its design opening limit at 0° F if it were installed at a superstructure average temperature of 80° F. Expansion gap widths at temperatures other than the normal temperature are generally not specified on rapid cure silicone sealant retrofit plans.

C. Asphaltic Plug Joints

Asphaltic plug joints consist of a flexible polymer modified asphalt installed in a preformed block out atop a steel plate and backer rod. In theory, asphaltic plug joints provided a seamless smooth riding surface. However, when subjected to high traffic counts, heavy trucks, or substantial acceleration/deceleration traction, the polymer modified asphalt tends to creep, migrating out of the block outs. As a consequence, we no longer specify the use of asphaltic plug joints.
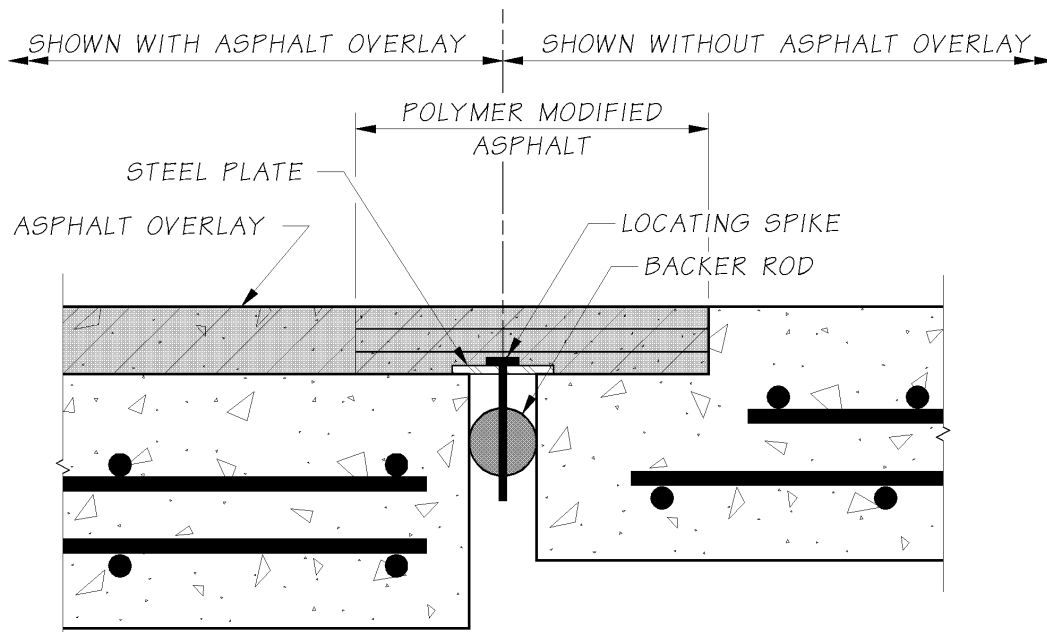
Asphaltic Plug Joint
Figure 9.1.3-3

D. Headers

Expansion joint headers for new construction are generally the same Class 4000D structural concrete as used for the bridge deck and cast integrally with the deck.

Expansion joint headers installed as part of a rehabilitative and/or overlay project are constructed differently.

Being a flexible material, hot mix asphalt (HMA) cannot provide rigid lateral support to an elastomeric compression seal or a rapid cure silicone sealant bead. Therefore, rigid concrete headers must be constructed on each side of such an expansion joint when an HMA overlay is installed atop an existing concrete deck. These headers provide a rigid lateral support to the expansion joint device and serve as a transition between the HMA overlay material and the expansion joint itself.

WSDOT allows either polyester concrete or elastomeric concrete for expansion joint headers. These two materials, which provide enhanced durability to impact in regard to other concrete mixes, shall be specified as alternates in the contract documents. Bridge Special Provisions (BSP)02206.GB6 and BSP023006.GB6 specify the material and construction requirements for polyester concrete. Bridge Special Provisions BSP02207.GB6 and BSP023007.GB6 specify the material and construction requirements for elastomeric concrete.

Modified concrete overlay (MCO) material can provide rigid side support for an elastomeric compression seal or a rapid cure silicone sealant bead without the need for separately constructed elastomeric concrete or polyester concrete headers. This alternative approach requires the approval of the Bearing and Expansion Joint Specialist. Such modified concrete overlay headers may utilize welded wire fabric as reinforcement. Contract 7108 which includes Bridges No. 90/565N&S and 90/566N&S is an example. BSP02313410.GB6 specifies the construction requirements for this approach, including the requirement for a temporary form to keep the joint open during placement of the MCO.

9.1.4   Medium Movement Range Joints

A. Steel Sliding Plate Joints

Two overlapping steel plates, one attached to the superstructure on each side of the joint, can be used to provide a smooth riding surface across an expansion joint. Unfortunately, steel sliding plates do not generally provide an effective barrier against intrusion of water and deicing chemicals into the joint and onto substructure elements. Consequently, these joints have been supplanted by newer systems, such as strip seals, with improved resistance to water penetration.
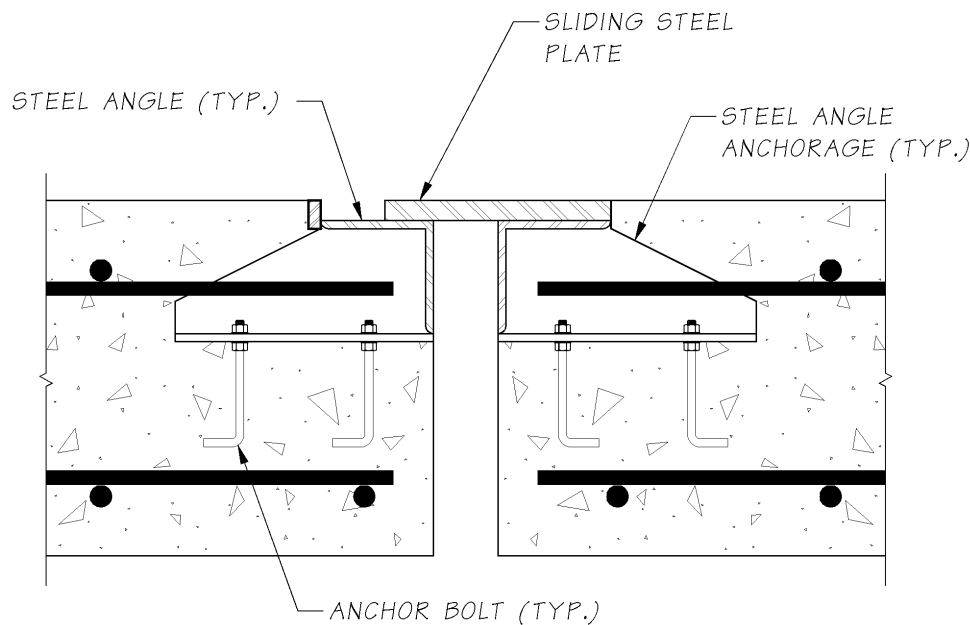
Steel Sliding Plate Joint
Figure 9.1.4-1

Before the advent of more modern systems, steel sliding plates were specified extensively. Their limited use today includes the following specific applications: 1) high pedestrian use sidewalks, 2) modular expansion joint upturns at traffic barriers, and 3) roadway applications involving unusual movements (translation and large rotations) not readily accommodated by modular expansion joints. In these applications, the sliding plates are generally galvanized or painted to provide corrosion resistance.

Repeated impact and corrosion have deteriorated many existing roadway sliding steel plate systems. In many instances, the anchorages connecting the sliding plate to the concrete deck have broken. When the integrity of the anchorages has been compromised, the steel sliding plates must generally be removed in their entirety and replaced with a new, watertight system. Where the integrity of the anchorages has not been compromised, sliding plates can often be retrofitted with poured sealants or elastomeric strip seals.

B. Strip Seal Joints

An elastomeric strip seal system consists of a preformed elastomeric gland mechanically locked into metallic edge rails generally embedded into the concrete deck on each side of an expansion joint gap. Unfolding of the elastomeric gland accommodates movement. Steel studs are generally welded to the steel extrusions constituting the edge rails to facilitate anchorage to the concrete deck. Damaged or worn glands can be replaced with minimal traffic disruption.

The metal edge rails effectively armor the edges of the expansion joint, obviating the need for a special impact resistant concrete, usually required at compression seal and poured sealant joints. The designer must select either the standard or special anchorage. The special anchorage incorporates steel reinforcement bar loops welded to intermittent steel plates, which in turn are welded to the extrusion. The special anchorage is generally used for very high traffic volumes or in applications subject to snowplow hits. In applications subject to snowplow hits and concomitant damage, the intermittent steel plates can be detailed to protrude slightly above the roadway surface in order to launch the snowplow blade and prevent it from catching on the forward extrusion.

The special anchorage requires a 9 in. deep block out, as opposed to 7 in. deep for the standard anchorage. The standard anchorage is acceptable for high traffic volume expansion joint replacement projects where block out depth limitations exist.
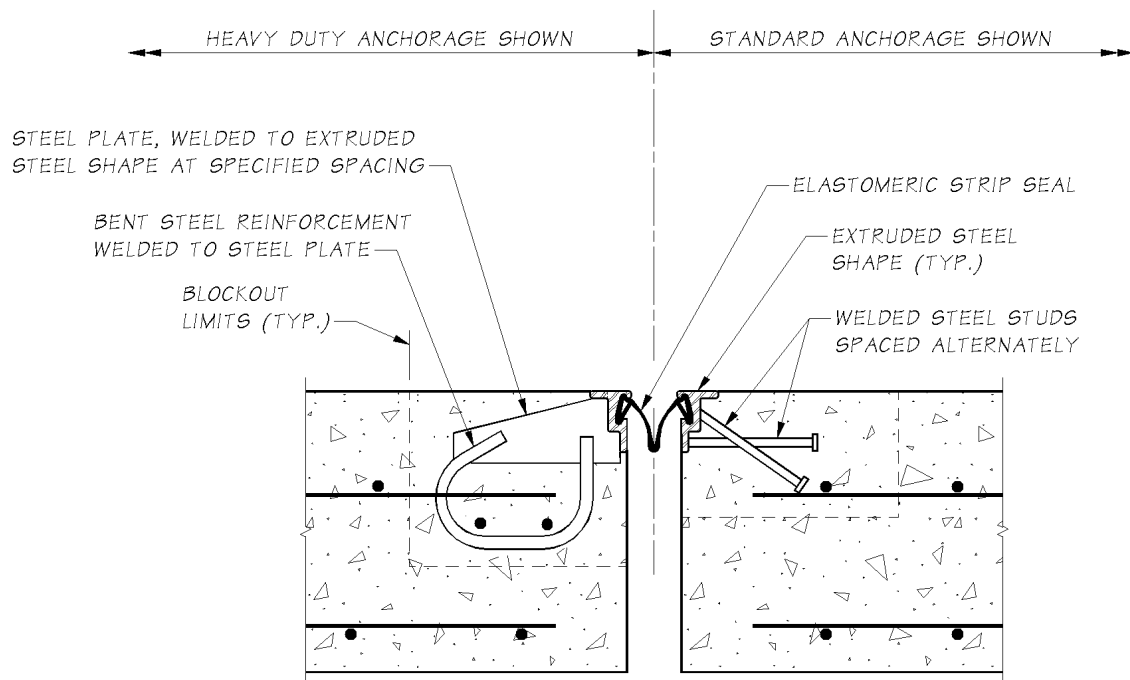
Strip Seal Joint
Figure 9.1.4-2

Design Example:

Given: A steel plate girder bridge has a total length of 600 ft. It is symmetrical and has a strip seal expansion joint at each end. These expansion joints are skewed 10°. Interior piers provide negligible restraint against longitudinal translation. Bridge superstructure average temperatures are expected to range between –30° F and 120° F during the life of the bridge. Assume a normal installation temperature of 64° F.

Find: Required Type A and Type B strip seal sizes and construction gap widths at 40° F, 64° F, and 80° F. Type A strip seals have a ½ in. gap at full closure. Type B strip seals are able to fully close, leaving no gap.

Solution:

Step 1: Calculate temperature and shrinkage movement.

Temperature: Δ_{temp} = ½(.0000065)(150° F)(600′)(12″/′) = 3.51″

Shrinkage: Δ_{shrink} = 0.0 (no shrinkage; μ = 0.0 for steel bridge)

Total deck movement at each joint: = 3.51″

$\Delta_{temp-normal-closing}$ = (120° F - 64° F)/(120° F + 30° F)(3.51″)(cos 10°)
                = 1.29″

$\Delta_{temp-normal-opening}$ = (64° F + 30° F)/(120° F + 30° F)(3.51″)(cos 10°)
                = 2.17″

Step 2: Determine strip seal size required. Assume a minimum construction gap width of 1½″ at 64° F.

Type A: Construction gap width of 1½″ at 64° F will not accommodate 1.29″ closing with a ½″ gap at full closure. Therefore, minimum construction gap width at 64° F must be 1.29″ + 0.50″ = 1.79″

   Size required = 1.79″ + 2.17″ = 3.96″ →Use 4″ strip seal

Type B: Construction width of 1½″ at 64° F is adequate.

   Size required = 1.50″ + 2.17″ = 3.67″ →Use 4″ strip seal

Step 3: Evaluate construction gap widths for various temperatures for a 4″ strip seal.

Type A: Required construction gap width at 64° F   = 0.50″ + 1.29″ = 1.79″

   Construction gap width at
   40° F = 1.79″ + (64° F - 40°F)/(64° F + 30° F)·(2.17″) = 2.34″

   Construction gap width at
   80°F = 1.79″ – (80° F – 64° F)/(120°F – 64° F)·(1.29″)=1.42″

Type B: Construction gap width of 1½″ at 64°F is adequate.

   Construction gap width at
   40° F = 1.50″ + (64° F - 40°F)/(64° F + 30° F)·(2.17″) = 2.05″

   Construction gap width at
   80° F = 1.50″ – (80° F – 64° F)/(120° F – 64° F)·(1.29″) = 1.13″

Conclusion: Use a 4 <u>in.</u> strip seal. Construction gap widths for installation at superstructure average temperatures of 40° F, 64° F, and 80° F are 2-5/16″, 1-13/16″, and 1-7/16″ for Type A and 2-1/16″, 1½″, and 1⅛″ for Type B. (Note that slightly larger gap settings could be specified for the 4″ Type B strip seal in order to permit the elastomeric glands to be replaced at lower temperatures at the expense of ride smoothness across the joint.)

C. Bolt-down Panel Joints

   Bolt-down panel joints, sometimes referred to as expansion dams, are preformed elastomeric panels internally reinforced with steel plates. Bridging across expansion gaps, these panels are bolted into formed block outs in the concrete deck with either adhesive or expansive anchors. Expansion is accompanied by stress and strain across the width of the bolt-down panel between anchor bolts.
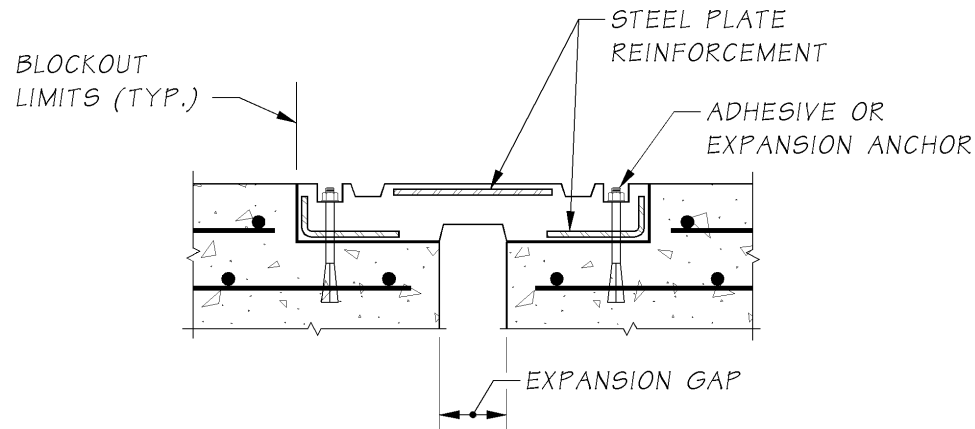
Bolt-down Panel Joint
Figure 9.1.4-3

Unfortunately, the anchorages are prone to loosening and breaking out under high-speed traffic. The resulting loose panels and hardware in the roadway present hazards to vehicular traffic, particularly motorcycles. As a consequence of the increased liability, we no longer specify bolt-down panel joints. On bridge overlay and expansion joint rehabilitation projects, bolt-down panels are being replaced with rapid-cure silicone sealant joints or strip seal joints. For rehabilitation of bridges having low speed or low volume traffic, existing bolt-down panel joints may be retained and/or selective damaged panels replaced.

9.1.5   Large Movement Range Joints

A   Steel Finger Joints

Finger joints have been successfully used to accommodate medium and large movement ranges. They are generally fabricated from steel plate and are installed in cantilevered configurations. The steel fingers must be designed to support traffic loads with sufficient stiffness to preclude excessive vibration. In addition to longitudinal movement, finger joints must also accommodate any rotations or differential vertical deflection across the joint. Finger joints may be fabricated with a slight downward taper toward the ends of the fingers in order to minimize potential for snowplow blade damage. Unfortunately, finger joints do not provide an effective seal against water infiltration. Elastomeric and metal troughs have been installed beneath steel finger joints to catch and redirect runoff water. However, in the absence of routine maintenance, these troughs clog and become ineffective.
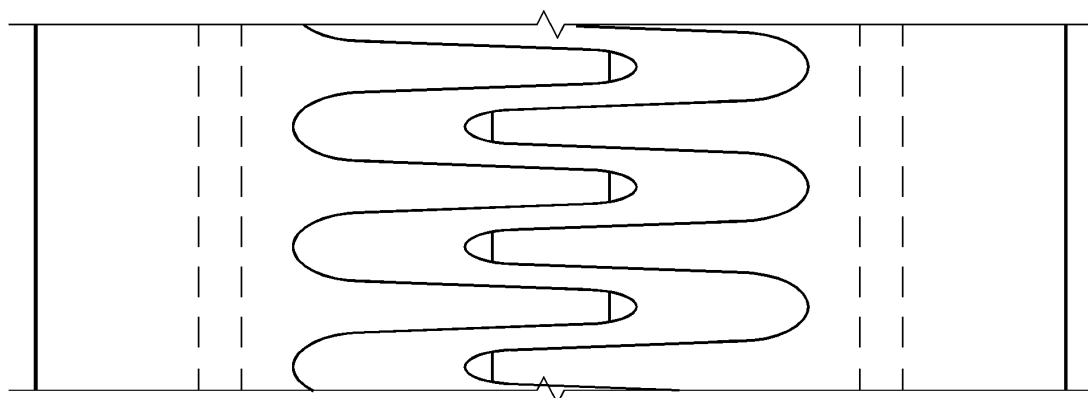
Steel Finger Joint
Figure 9.1.5-1

B. Modular Expansion Joints

Modular expansion joints are complex structural assemblies designed to provide watertight wheel load transfer across expansion joint openings. These systems were developed in Europe and introduced into the U.S. in the 1960s. To date, modular expansion joints have been designed and fabricated to accommodate movements of up to 85 in. In Washington state, the largest modular expansion joints are those on the new Tacoma Narrows Bridge. These joints accommodate 48 in. of service movement and 60 in. of seismic movement. Modular expansion joints are generally shipped in a completely assembled configuration. Although center beam field splices are not preferable, smaller motion range modular expansion joints longer than 40 ft. may be shipped in segments to accommodate construction staging and/or shipping constraints.

1. Operational Characteristics

Modular expansion joints comprise a series of steel center beams oriented parallel to the expansion joint axis. Elastomeric strip seals or box-type seals attach to adjacent center beams, preventing infiltration of water and debris. The center beams are supported on support bars, which span in the primary direction of anticipated movement. The support bars are supported on sliding bearings mounted within support boxes. Polytetrafluoroethylene (PTFE) - stainless steel interfaces between elastomeric support bearings and support bars facilitate the unimpeded translation of the support bars as the expansion gap opens and closes. The support boxes generally rest on either cast-in-place concrete or grout pads installed into a preformed block out.

Modular expansion joints can be classified as either single support bar systems or multiple support bar systems. In multiple support bar systems, a separate support bar supports each center beam. In the more complex single support bar system, one support bar supports all center beams at each support location. This design concept requires that each center beam be free to translate along the longitudinal axis of the support bar as the expansion gap varies. This is accomplished by attaching steel yokes to the underside of the center beams. The yoke engages the support bar to facilitate load transfer. Precompressed elastomeric springs and PTFE – stainless steel interfaces between the underside of each center beam and the top of the support bar and between the bottom of the support bar and bottom of the yoke support each center beam and allow it to translate along the longitudinal axis of the support bar. Practical center beam span lengths limit the use of multiple support bar systems for larger movement range modular expansion joints. Multiple support bar systems typically become impractical for more than nine seals or for movement ranges exceeding 27″. Hence, the single support bar concept typifies these larger movement range modular expansion joints.
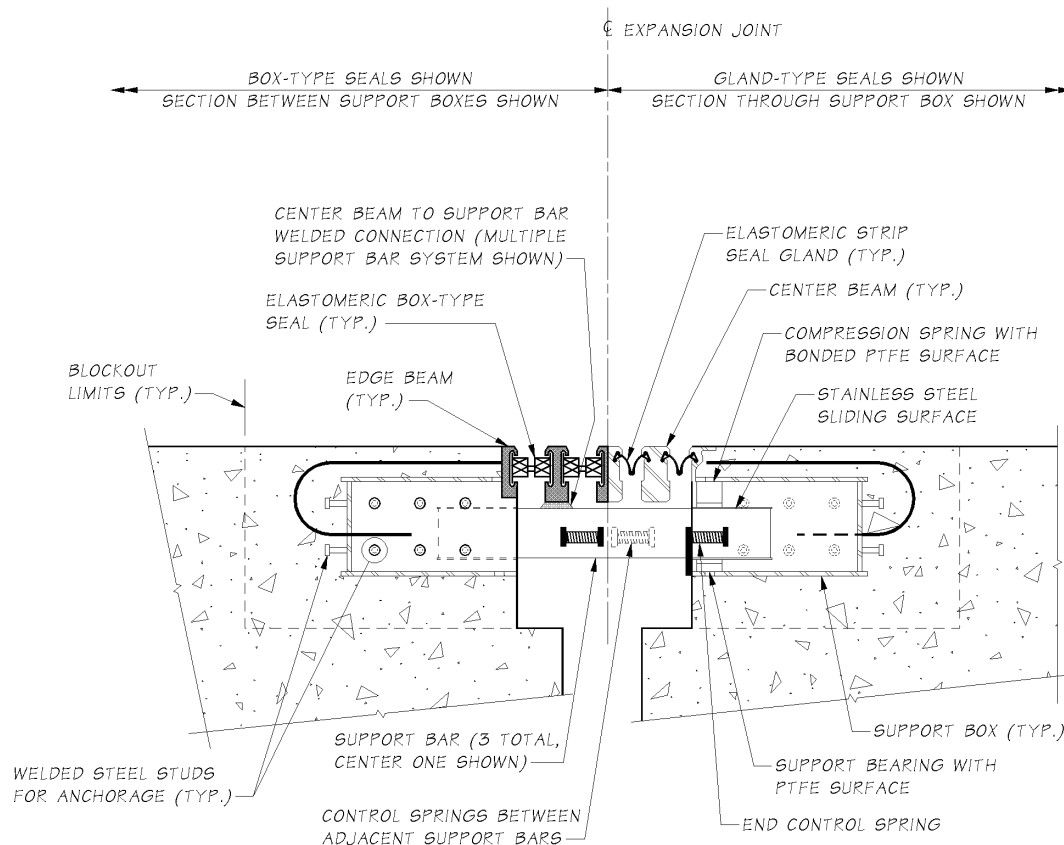
Modular Expansion Joint
Figure 9.1.5-2

The highly repetitive nature of axle loads predisposes modular expansion joint components and connections to fatigue susceptibility, particularly at center beam to support bar connections and center beam field splices. Bolted connections of center beams to support bar have demonstrated poor fatigue endurance. Welded connections are preferred, but must be carefully designed, fatigue tested, fabricated, and inspected to assure satisfactory fatigue resistance. WSDOT'S current special provision for modular expansion joints requires stringent fatigue based design criteria for modular expansion joints. This special provision also specifies criteria for manufacturing, shipping, storing, and installing modular expansion joints.

Modular expansion joints may need to be shipped and/or installed in two or more pieces and subsequently spliced together in order to accommodate project staging and/or practical shipping constraints. Splicing generally occurs after concrete is cast into the block outs. The center beams are the elements that must be connected. These field connections are either welded, bolted, or a hybrid combination of both.

Center beam field splices have historically been the weak link of modular expansion joints because of their high fatigue susceptibility and their tendency to initiate progressive zipper-type failure. The reduced level of quality control achievable with a field operation in regard to a shop operation contributes to this susceptibility. Specific recommendations regarding center beam field splices will be subsequently discussed as they relate to shop drawing review and construction.

2. Movement Design

Calculated total movement range establishes modular expansion joint size. WSDOT policy has been to provide a 15 <u>percent</u> factor of safety on these calculated service movements. Current systems permit <u>approximately</u> 3 <u>in.</u> of movement per elastomeric seal element; hence total movement rating provided will be a multiple of 3 <u>in</u>. To minimize impact and wear on bearing elements, the maximum gap between adjacent center beams should be limited to about 3½ <u>in</u>.

To facilitate the installation of the modular joints at temperatures other than the 64° F normal temperature, the contract drawings shall specify expansion gap distance face-to-face of edge beams as a function of the superstructure temperature at the time of installation.

Modular expansion joint movement design relationships can be expressed as:

$$n = MR / mr$$
$$G_{min} = (n-1) \cdot w + n \cdot g$$
$$G_{max} = G_{min} + M7R$$

where MR = total movement range of the modular joint

mr   =   movement range per elastomeric seal

n   =   number of seals

n – 1 =   number of center beams

w   =   width of each center beam

g   =   minimum gap per strip seal element at full closure

G_{min} =   minimum distance face-to-face of edge beams

G_{max} =   maximum distance face-to-face of edge beams

Design Example:

Given: Two cast-in-place post-tensioned concrete box girder bridge frames meet at an intermediate pier where they are free to translate longitudinally. Skew angle is 0° and the bridge superstructure average temperature ranges from 0° F to 120° F. A modular bridge expansion joint will be installed 60 days after post-tensioning operations have been completed. Specified creep is 150 <u>percent</u> of elastic shortening. Assume that 50 <u>percent</u> of total shrinkage has already occurred at installation time. The following longitudinal movements were calculated for each of the two frames:

	Frame A	Frame B
Shrinkage	1.18″	0.59″
Elastic shortening	1.42″	0.79″
Creep (1.5 × Elastic shortening)	2.13″	1.18″
Temperature fall (64° F to 0° F)	3.00″	1.50″
Temperature rise (64° F to 120° F)	2.60″	1.30″

Find: Modular expansion joint size required to accommodate the total calculated movements and the installation gaps measured face-to-face of edge beams at superstructure average temperatures of 40° F, 64° F, and 80° F.

Solution:

Step 1: Determine modular joint size.

Total opening movement (Frame A)	$= (0.5)\cdot(1.18'') + 2.13'' + 3.00''$
	$= 5.72''$
Total opening movement (Frame B)	$= (0.5)\cdot(0.59'') + 1.18'' + 1.50''$
	$= 2.98''$
Total opening movement (both frames)	$= 5.72'' + 2.98'' = 8.70''$
Total closing movement (both frames)	$= 2.60'' + 1.30'' = 3.90''$

Determine size of the modular joint, including a 15 <u>percent</u> allowance:

$1.15\cdot(8.70'' + 3.90'') = 14.49'' \rightarrow$ Use a 15 <u>in.</u> movement rating joint

Step 2: Evaluate installation gaps measured face-to-face of edge beams at superstructure average temperatures of 40° F, 64° F, and 80° F.

$MR = 15''$ (movement range)

$mr = 3''$ (maximum movement rating per strip seal element)

$n = 15''/3'' = 5$ strip seal elements

$n - 1 = 4$ center beams

$w = 2.50''$ (center beam top flange width)

$g = 0''$

$G_{min} = 4\cdot(2.50'') + 4\cdot(0'') = 10''$

$G_{max} = 10'' + 15'' = 25''$

$G_{64F} = G_{min} +$ Total closing movement from temperature rise

$\qquad = 10'' + 1.15\cdot(3.90'') = 14.48'' \rightarrow$ Use 14½''

$G_{40F} = 14.5'' + [(64° F-40° F)/(64° F-0° F)]\cdot(3.00'' + 1.50'') = 16.19''$

$G_{80F} = 14.5'' - [(80° F-64° F)/(120° F-64° F)]\cdot(2.60'' + 1.30'') = 13.39''$

Check spacing between center beams at minimum temperature:

$G_{0F} = 14.50'' + 8.70'' = 23.20''$

Spacing $= [23.20'' - 4(2.50'')]/5 = 2.64'' < 3½'' \rightarrow$ OK

Check spacing between center beams at 64° F for seal replacement:

Spacing $= [14.50'' - 4(2.50'')]/5 = 0.90'' < 1.50''$ Therefore, the center beams must be mechanically separated in order to replace strip seal elements.

Conclusion: Use a 15 <u>in.</u> modular expansion joint. The gaps measured face-to-face of edge beams at installation temperatures of 40° F, 64° F, and 80° F are 16-3/16 <u>in.</u>, 14½ <u>in.</u> and 13⅜ <u>in.</u>, respectively.

3. Review of Shop Drawings and Structural Design Calculations

The manufacturer's <u>e</u>ngineer generally performs structural design of modular expansion joints. The project special provisions require<u>s</u> that the manufacturer submit structural calculations, detailed fabrication drawings, and applicable fatigue tests for approval by the Engineer. All structural elements must be designed and detailed for both strength and fatigue. Additionally, modular expansion joints should be detailed to provide access for inspection and periodic maintenance activities, including replacement of seals, control springs, and bearing components.

WSDOT's special provision for modular expansion joints delineates explicit requirements for their design, fabrication, and installation. This comprehensive special provision builds upon WSDOT's past experience specifying modular expansion joints and incorporates the NCHRP Report 402 *Fatigue Design of Modular Bridge Expansion Joints*. The special provisions include requirements for the shop drawings, calculations, material certifications, general fabrication methods, corrosion protection, shipping and handling, storage, installation, fatigue testing, applicable welding codes and certifications, quality control, and quality assurance. It is strongly advised to carefully review this special provision before reviewing modular expansion joint shop drawings and calculations.

Any structural details, including connections, that do not clearly correspond to specific fatigue categories depicted in the LRFD shall be fatigue tested in accordance with the requirements stipulated in the special provision. Documentation of these tests shall accompany the shop drawing submittal.

As stated in the special provisions, the Contractor shall submit documentation of a quality assurance program distinctly separate from in-house quality control. Quality assurance shall be performed by an independent agency and shall be provided by the manufacturer.

Weld procedures shall be submitted for all shop and field welds. These procedures stipulate welding process employed, end preparation of the component welded, weld metal type, preheat temperature, and welder certifications. It is critical that all welds be made in strict accordance with specifications and under very careful inspection.

Field splices of center beams require particularly careful review. WSDOT's special provision recommends several mitigating measures to minimize fatigue susceptibility of center beam field splices. These measures include reducing support box spacing and optimizing fatigue stress range at field splice locations. Keep in mind that the confined nature of the space in which a welder must work can make these welds very difficult to complete. The American Welding Society (AWS) Welding Code prequalifies certain end geometries because experience has shown that high quality welds can be achieved.

Non-prequalified center beam end geometries require the Contractor to submit a Procedure Qualification Record documenting that satisfactory weld quality has been achieved using samples before welding of the actual field piece. The Contractor will generally want to avoid the additional expense associated with these tests and will thus specify a prequalified end geometry.

WSDOT's special provisions require that adequate concrete consolidation be achieved underneath all support boxes. The reviewer should ascertain that the shop drawings detail a vertical minimum of 2 in. between the bottom of each support box and the top of the concrete block out. Alternatively, when vertical clearance is minimal, grout pads can be cast underneath support boxes before casting the concrete within the blockout.

4. Construction Considerations

Temperature adjustment devices are temporarily welded to the modular expansion joints to permit the Contractor to adjust the modular joint width so that it is consistent with the superstructure temperature at the time concrete is placed in the block out. The temperature devices effectively immobilize the modular joint. Once the concrete begins to set up, it is critical to remove these devices as soon as possible. If the modular expansion joint is prevented from opening and closing, it will be subject to very large, potentially damaging, forces.

Prior to placement of concrete into the block out, temporary supports <u>generally bridge</u> across the expansion gap, suspend<u>ing</u> the modular <u>expansion</u> joint from the bridge deck surface. Following concrete placement, the modular joint is supported by bearing of the support boxes on concrete that has consolidated underneath the blockout. The inspector should assure that adequate concrete consolidation is achieved underneath and around the support boxes.

Following delivery of the modular <u>expansion</u> joint to the jobsite and prior to <u>its</u> installation, the inspector should ascertain that center beam end geometries at field weld splice locations match those shown on the approved weld procedure.

9.2   Bearings

9.2.1   General Considerations

Bridge bearings facilitate the transfer of vehicular and other environmentally imposed loads from the superstructure down to the substructure, and ultimately, to the ground. In fulfilling this function, bearings must accommodate anticipated movements (thermal expansion/contraction) while also restraining undesired movements (seismic displacements). Because the movements allowed by an adjacent expansion joint must be compatible with the movement restriction imposed by a bearing, bearings and expansion joints must be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

Numerous types of bearings are used for bridges. These include steel reinforced elastomeric bearings, fabric pad sliding bearings, steel pin bearings, rocker bearings, roller bearings, steel pin bearings, pot bearings, spherical bearings, disk bearings, and seismic isolation bearings. Each of these bearings possess different characteristics <u>in regard to</u> vertical and horizontal load carrying capacity, vertical stiffness, horizontal stiffness, and rotational stiffness. A thorough understanding of these characteristics is essential for economical bearing selection and design. Spherical bearings, disk bearings, and pot bearings are sometimes collectively referred to as high load multi-rotational (HLMR) bearings.

9.2.2   Force Considerations

Bridge bearings must be explicitly designed to transfer all anticipated loads from the superstructure to the substructure. These forces may be directed vertically, longitudinally, or transversely with respect to the global orientation of the bridge. In accordance with LRFD provisions, most bearing design calculations are based upon service limit state stresses. Impact need not be applied to live load forces in the design of bearings.

Experience has empirically led to the following practical load capacity approximations for various bearing types:

Bearing Type	Approx. Load Capacity
Steel reinforced elastomeric (Method B)	Less than 800 kips
Fabric pad	Less than 600 kips
Steel pin	More than 600 kips
Spherical and disk	More than 800 kips
Seismic isolation	Less than 800 kips

9.2.3   Movement Considerations

Bridge bearings can be detailed to provide translational fixity, to permit free translation in any horizontal direction, or to permit guided translation. The movement restrictions thus imposed by a bearing must be compatible with the movements allowed by an adjacent expansion joint. Additionally, both bearings and expansion joints must be designed consistent with the anticipated load and deformation behavior of the overall structure. Design rotations shall be calculated as follows:

A. Elastomeric and Fabric Pad Bearings

The maximum service limit state rotation for bearings that do not have the potential to achieve hard contact between metal components shall be taken as the sum of unfactored dead and live load rotations plus an allowance for fabrication and construction uncertainties of 0.005 radians.

B. HLMR Bearings

The maximum strength limit state load rotation for bearings that are subject to potential hard contact between metal components shall be taken as the sum of all applicable factored load rotations plus an allowance of 0.01 radians for fabrication and installation tolerances and an allowance of 0.01 radians for other uncertainties. Such bearings include spherical, pot, steel pin, and some types of seismic isolation bearings.

9.2.4   Detailing Considerations

HLMR bearings shall be designed, detailed, fabricated, and installed to facilitate inspection, maintenance, and eventual replacement. Jacking points shall be identified in the contract drawings so that bearings can be reset, repaired, or replaced. In some instances, bearings may need to be reset in order to mitigate unintended displacements induced by construction sequences.

Prestressed concrete girder bridges having end Type A (semi - integral) need not be detailed to accommodate elastomeric bearing replacement at abutments. Prestressed concrete girder bridges having end Type B (L-type abutments) shall be designed and detailed to accommodate elastomeric bearing replacement at abutments. Specifically, girder stops and end diaphragms shall be detailed to accommodate the placement of hydraulic jacks. The standard end diaphragms for long-span girders may not have sufficient flexural and shear capacity to support jacking induced stresses. The designer shall check these and provide sufficient steel reinforcement to accommodate shear forces and bending moments induced by jacking. (Girder end Types A and B are depicted on Figures 5.6.2-4 and 5.6.2-5.) Incidentally, intermediate piers of prestressed concrete girder bridges having steel reinforced elastomeric bearings shall also be designed and detailed to facilitate bearing replacement.

9.2.5   Bearing Types

A. Elastomeric Bearings

Elastomeric bearings are perhaps the simplest and most economical of all bridge bearings. They are broadly classified into four types: plain elastomeric pads, fiberglass reinforced elastomeric pads, steel reinforced elastomeric pads, and cotton duck reinforced elastomeric pads. Of these four types, the latter two are used extensively for bridge construction. Incidentally, cotton duck reinforced elastomeric pads are generally referred to as fabric pad bearings. This subsection will address steel reinforced elastomeric bearings. A subsequent section will address fabric pad bearings.

A steel reinforced elastomeric bearing consists of discrete steel shims vulcanized between adjacent discrete layers of elastomer. The vulcanization process occurs in an autoclave under conditions of high temperature and pressure. The constituent elastomer is either natural rubber or synthetic rubber (neoprene). Steel reinforced elastomeric bearings are commonly used with prestressed concrete girder bridges and may be used with other bridge types. Because of their relative simplicity and fabrication ease, steel reinforced elastomeric bearings offer significant economy relative to HLMR bearings.

Steel reinforced elastomeric bearings rely upon the inherent shear flexibility of the elastomer layers to accommodate bridge movements in any horizontal direction. This shear flexibility also enhances their rotational flexibility. The steel shims limit the tendency for the elastomer layers to bulge laterally under compressive load.

Steel reinforced elastomeric bearings can be designed by either the Method A or Method B procedure delineated in the LRFD provisions. Current WSDOT policy is to design all elastomeric bearings using the Method B provisions, which provides more relief in meeting rotational demands than Method A. The Method A design procedure is a carryover based

upon more conservative interpretation of past theoretical analyses and empirical observations prior to research leading up to the publication of *NCHRP Report 596 Rotation Limits for Elastomeric Bearings.*

Both Method A and Method B design procedures require determination of the optimal geometric parameters to achieve an appropriate balance of compressive, shear, and rotational stiffnesses and capacities. Fatigue susceptibility is controlled by limiting live load compressive stress. Delamination (of steel shim-elastomer interface) susceptibility is controlled by limiting total compressive stress. Assuring adequate shim thickness precludes yield and rupture of the steel shims. Excessive shear deformation is controlled and rotational flexibility is assured by providing adequate total elastomer height. Generally, total elastomer thickness shall be no less than twice the maximum anticipated lateral deformation. Overall bearing stability is controlled by limiting total bearing height relative to its plan dimensions. The most important design parameter for reinforced elastomeric bearings is the shape factor. The shape factor is defined as the plan area of the bearing divided by the area of the perimeter free to bulge (perimeter multiplied by thickness of one layer of elastomer).

Axial, rotational, and shear loading generate shear strain in the constituent elastomeric layers of a typical bearing. Computationally, Method B imposes a limit on the sum of these shear strains. It distinguishes between static and cyclic components of shear strain by applying an amplification factor of 1.75 to cyclic components to reflect cumulative degradation caused by repetitive loading.

In essence, elastomeric bearing design reduces to checking several mathematical equations while varying bearing plan dimensions, number of elastomeric layers and their corresponding thicknesses, and steel shim thicknesses. Because these calculations can become rather tedious, MS Excel spreadsheets have been developed and are available for designs using both Method A and Method B procedures. See the Bearing and Expansion Joint Specialist for these design tools.

LRFD design <u>may</u> result in thicker steel reinforced elastomeric bearings than previous designs, particularly for shorter span bridges. This is <u>a consequence of the increased rotational flexibility required to accommodate the 0.005 radian allowance for uncertainties and partially to inherent conservatism built into the rotational capacity equations.</u>

Although constituent elastomer has historically been specified by durometer hardness, shear modulus is the most important physical property of the elastomer for purposes of bearing design. Research has concluded that shear modulus may vary significantly among compounds of the same hardness. Accordingly, shear modulus shall be specified on the plans as 165 psi at 73º F without reference to durometer hardness.

Elastomeric bearings shall conform to the requirements of AASHTO Specification M 251 *Plain and Laminated Elastomeric Bridge Bearings*. Shims shall be fabricated from ASTM A 1011 Grade 36 steel unless noted otherwise on the plans. Bearings shall be laminated in ½ inch thick elastomeric layers with a minimum total thickness of 1 inch. For overall bearing heights less than <u>5</u> inches, a minimum of ¼ inch of side clearance shall be provided over the steel shims. For overall heights greater than <u>5</u> inches, a minimum of ½ inch of side clearance shall be provided. Live load compressive deflection shall be limited to 1/16 inch. AASHTO Specification M 251 requires elastomeric bearings to be subjected to a series of tests, including a compression test at 150 <u>percent</u> of the total service load. For this reason, compressive dead load and live load shall be specified on the plans.

With respect to width, elastomeric bearings shall be designed and detailed as follows:

1. For prestressed concrete wide flange girders (WF42G, WF50G, WF58G, WF74G, and W95G), the edge of the bearing pad shall be set between 1 in. minimum and 9 in. maximum inside of the edge of the girder bottom flange.

2. For prestressed concrete I-girders, bulb-tee girders, and deck bulb-tee girders, the edge of the bearing pad shall be set 1 in. in side of the edge of the girder bottom flange.

3. For all prestressed concrete tub girders, the edge of the bearing shall be set 1 in. inside of the edge of the bottom slab. Bearing pads for prestressed concrete tub girders shall be centered close to the centerline of each web.

4. For all prestressed concrete slabs, the edge of the bearing shall be set 1 in. inside of the edge of the slab. Two bearing pads and corresponding grout pads are required for each end of the prestressed concrete slabs. The need for steel shims shall be assessed during the bearing design.

As mentioned earlier, LRFD Article 14.4.2.1 requires that a 0.005 radian allowance for uncertainties be included in the design of steel reinforced elastomeric bearings. This allowance applies to both rotations θ_x and θ_y. The Article 14.4.2 Commentary somewhat ambiguously states "An owner may reduce the fabrication and setting tolerance allowances if justified by a suitable quality control plan; therefore, these tolerance limits are stated as recommendations rather than absolute limits." Consult with the Bearings and Expansion Joint Specialist in instances in which the 0.005 radian tolerance precludes convergence to a reasonable design solution.

In order to facilitate compressive load testing, future bearing replacement, and vertical geometry coordination, the following table shall be included in the Plans:

Bearing Design Table	
Service I Limit State	
Dead load reaction	--------- kips
Live load reaction (w/o impact)	--------- kips
Unloaded height	--------- inches
Loaded height (DL)	--------- inches
Shear modulus at 73° F	--------- psi

In the construction of precast prestressed concrete girder and steel girder bridges, elastomeric bearings are generally not offset to account for temperature during erection of the girders as are most other bearing systems. Girders may be set atop elastomeric bearings at temperatures other than the mean of the temperature range. This is statistically reconciled by assuming a maximum thermal movement in either direction of:

$$\Delta_{temp} = 0.75 \cdot \alpha \cdot L \cdot (T_{MaxDesign} - T_{MinDesign})$$

where $T_{MaxDesign}$ is the maximum anticipated bridge deck average temperature and $T_{MinDesign}$ is the minimum anticipated bridge deck average temperature during the life of the bridge.

For precast prestressed concrete girder bridges, the maximum thermal movement, Δ_{temp}, shall be added to shrinkage and long-term creep movements to determine total bearing height required. The shrinkage movement for this bridge type shall be half that calculated for a cast-in-place concrete bridge.

For cast-in-place concrete bridges, it is assumed that the temperature of concrete at placement is equal to the normal temperature, as defined by the *Standard Specifications*. Total shrinkage movement is added to the maximum thermal movement, Δ_{temp}, to determine required total height of the elastomeric bearing, as noted in Section 9.1.2-A.

B. Fabric Pad Sliding Bearings

Fabric pad sliding bearings incorporate fabric pads with a polytetrafluoroethylene (PTFE) - stainless steel sliding interface to permit large translational movements. Unlike a steel reinforced elastomeric bearing having substantial shear flexibility, the fabric pad alone cannot accommodate translational movements. Fabric pads can accommodate very small amounts of rotational movement; less than can be accommodated by more flexible steel reinforced elastomeric bearings. Practical size considerations limit the use of fabric pad bearings to total service load reactions under about 600 kips.

PTFE, also referred to as Teflon, is available in several forms: unfilled sheet, dimpled lubricated, filled, and woven. Filled PTFE contains glass, carbon, or other chemically inert fibers that enhance its resistance to creep (cold flow) and wear. Interweaving high strength fibers through PTFE material creates woven PTFE. Dimpled PTFE contains dimples, which act as reservoirs for silicone grease lubricant.

Friction coefficients for PTFE – stainless steel surfaces vary significantly as a function of PTFE type, contact pressure, and ambient temperature. The AASHTO LRFD provides friction coefficients as a function of these variables. Dimpled lubricated PTFE at high temperatures and high contact pressures typically yield the lowest friction coefficients. Filled PTFE at low temperatures and low contact pressures yield the highest friction coefficients.

In order to minimize frictional resistance, a Number 8 (Mirror) finish should be specified for all flat stainless steel surfaces in contact with PTFE. The low-friction characteristics of a PTFE – stainless steel interface are actually facilitated by fragmentary PTFE sliding against PTFE after the fragmentary PTFE particles are absorbed into the asperities of the stainless steel surface.

In fabric pad sliding bearings, the PTFE is generally recessed half its depth into a steel backing plate, which is generally bonded to the top of a fabric pad. The recess provides confinement that minimizes creep (cold flow). The stainless steel sheet is typically seal welded to a steel sole plate attached to the superstructure.

Silicone grease is not recommended for non-dimpled PTFE. Any grease will squeeze out under high pressure and attract potentially detrimental dust and other debris.

1. Fabric Pad Design

    WSDOT's design criteria for fabric pad bearings are based upon manufacturers' recommendations, supported by years of satisfactory performance. These criteria differ from AASHTO LRFD provisions in that they recognize significantly more rotational flexibility in the fabric pad. Our maximum allowable service load average bearing pressure for fabric pad bearing design is 1,200 psi. WSDOT's maximum allowable service load edge bearing pressure for fabric pad bearing design is 2,000 psi. A 1,200 psi compressive stress corresponds to 10 percent strain in the fabric pad while a 2,000 psi compressive stress corresponds to 14 percent compressive strain. Based upon this information, the following design relationship can be established:

    $$\theta = \frac{2 \times (.14 - .10) \times T}{L}$$
    $$\theta = \frac{.08 \times T}{L}$$
    $$T = 12.5 \times \theta \times L$$

where Θ = rotation due to loading plus construction tolerances

L = pad length (parallel to longitudinal axis of beam)

T = fabric pad thickness required

As an example:

Given: DL + LL = 240 kips

Rotation = 0.015 radians

Allowable bearing pad pressure = 1200 psi

$f_c' = 3000$ psi

Find: fabric pad plan area and thickness required

Solution:

Pad area required = 240,000/1200 = 200 in²

Try a 20″ wide × 10″ long fabric pad

T = 12.5(.015)(10″) = 1.88″

Solution: Use a 20″ × 10″ × 1⅞″ fabric pad.

2.   PTFE – Stainless Steel Sliding Surface Design

PTFE shall be ⅛ in. thick and recessed 1/16 in. into a ½ in. thick steel plate that is bonded to the top of the fabric pad. With the PTFE confined in this recess, the LRFD code permits an average contact stress of 4,500 psi for all loads calculated at the service limit state and an average contact stress of 3,000 psi for permanent loads calculated at the service limit state. The LRFD code permits slightly higher edge contact stresses.

For example, suppose:

DL = 150 kips

LL = 90 kips

$A_{PTFE} > (150 \text{ kips} + 90 \text{ kips})/4.5 \text{ ksi} = 53.3$ in²

$A_{PTFE} > 150 \text{ kips}/3 \text{ ksi} = 50.0$ in²

Selected area of PTFE must exceed 53.3 in²

Stainless steel sheet shall be finished to a No. 8 (Mirror) finish and seal welded to the sole plate.

C.   Pin Bearings

Steel pin bearings are generally used to support heavy reactions with moderate to high levels of rotation about a single predetermined axis. This situation generally occurs with long straight steel plate girder superstructures.

D.   Rocker and Roller Type Bearings

Steel rocker bearings have been used extensively in the past to allow both rotation and longitudinal movement while supporting large loads. Because of their seismic vulnerability and the more extensive use of steel reinforced elastomeric bearings, rocker bearings are no longer specified for new bridges.

Steel roller bearings have also been used extensively in the past. Roller bearings permit both rotation and longitudinal movement. Pintles are generally used to connect the roller bearing to the superstructure above and to the bearing plate below. Nested roller bearings have also been used in the past. Having been supplanted by more economical steel reinforced elastomeric bearings, roller bearings are infrequently used for new bridges today.

E. Spherical Bearings

A spherical bearing relies upon the low-friction characteristics of a curved PTFE - stainless steel interface to provide a high level of rotational flexibility in multiple directions. An additional flat PTFE - stainless steel surface can be incorporated into the bearing to additionally provide either guided or non-guided translational movement capability.

Woven PTFE is generally used on the curved surfaces of spherical bearings. Woven PTFE exhibits enhanced creep (cold flow) resistance and durability characteristics relative to unwoven PTFE. When spherical bearings are detailed to accommodate translational movement, woven PTFE is generally specified on the flat sliding surface also. The LRFD code permits an average contact stress of 4,500 psi for all loads calculated at the service limit state and an average contact stress of 3,000 psi for permanent loads calculated at the service limit state. The LRFD code permits slightly higher edge contact stresses.

Both stainless steel sheet and solid stainless steel have been used for the convex sliding surface of spherical bearings. According to one manufacturer, curved sheet is generally acceptable for contact surface radii greater than 14 in. to 18. in.For smaller radii, a solid stainless steel convex plate or a stainless steel inlay is used. The inlay is welded to the solid conventional steel. If the total height of the convex plate exceeds about 5 in., a stainless steel inlay will likely be more economical.

Most spherical bearings are fabricated with the concave surface oriented downward to minimize dirt infiltration between PTFE and the stainless steel surface. Structural analysis of the overall structure must recognize the center of rotation of the bearing not being coincident with the neutral axis of the girder above.

The contract drawings must show the diameter and height of the spherical bearing in addition to all dead, live, and seismic loadings. Total height depends upon the radius of the curved surface, diameter of the bearing, and total rotational capacity required. Consult the Bearing and Expansion Joint Specialist for design calculation examples. Additionally, sole plate connections, base plate, anchor bolts, and any appurtenances for horizontal force transfer must be detailed on the plans. The spherical bearing manufacturer is required to submit shop drawings and detailed structural design calculations of spherical bearing components for review by the Engineer.

F. Disk Bearings

A disk bearing is composed of an annular shaped urethane disk designed to provide moderate levels of rotational flexibility. A steel shear-resisting pin in the center provides resistance against lateral force. A flat PTFE - stainless steel surface can be incorporated into the bearing to also provide translational movement capability, either guided or non-guided.

G. Seismic Isolation Bearings

Seismic isolation bearings mitigate the potential for seismic damage by utilizing two related phenomena: dynamic isolation and energy dissipation. Dynamic isolation allows the superstructure to essentially float, to some degree, while substructure elements below move with the ground during an earthquake. The ability of some bearing materials and elements to deform in certain predictable ways allows them to dissipate earthquake energy that might otherwise damage critical structural elements.

Numerous seismic isolation bearings exist, each relying upon varying combinations of dynamic isolation and energy dissipation. These devices include lead core elastomeric bearings, high damping rubber, friction pendulum, hydraulic dampers, and various hybrid variations.

Effective seismic isolation bearing design requires a thorough understanding of the dynamic characteristics of the overall structure as well as the candidate isolation devices. Isolation devices are differentiated by maximum compressive load capacity, lateral stiffness, lateral displacement range, maximum lateral load capacity, energy dissipation per cycle, functionality in extreme environments, resistance to aging, fatigue and wear properties, and effects of size.

The Highway Innovative Technology Evaluation Center (HITEC) has developed guidelines for testing seismic isolation and energy dissipating devices. With the goal of disseminating objective information to design professionals, HITEC has tested and published technical reports on numerous proprietary devices. These tests include performance benchmarks, compressive load dependent characterization, frequency dependent characterization, fatigue and wear, environmental aging, dynamic performance at extreme temperatures, durability, and ultimate performance.

9.2.6 Miscellaneous Details

A. Temporary Support before Grouting Masonry Plate

The masonry plate of a HLMR bearing is generally supported on a grout pad that is installed after the bearing and superstructure girders above have been erected. This procedure allows the Contractor to level and slightly adjust the horizontal location of the bearing before immobilizing it by placing the grout pad. Several methods have been developed to temporarily support the masonry plate until the grout is placed. The two most commonly used methods will be discussed here.

1. Shim Packs

Multiple stacks of steel shim plates can be placed atop the concrete surface to temporarily support the weight of the girders on their bearings before grouting. engineering judgment must be used in selecting the number and plan size of the shims taking grout flowability and shim height adjustability into consideration.

2. Two-step Grouting with Cast Sleeves

A two-step grouting procedure with cast-in-place voided cores can be used for smaller HLMRs not generally subjected to uplift. Steel studs are welded to the underside of the masonry plate to coincide with the voided cores. With temporary shims installed between the top of the concrete surface and the underside of the masonry plate, the voided cores are fully grouted. Once the first stage grout has attained strength, the shims are removed, the masonry plate is dammed, and grout is placed between the top of the concrete surface and the underside of the masonry plate.

B. Anchor Bolts

Anchor bolts shall be designed to resist all horizontal shear forces and direct tension force due to uplift.

Anchor bolts shall be ASTM A 449 where strengths equal to ASTM A 325 are required and ASTM A 354, Grade BD, where strengths equal to ASTM A 490 are required. ASTM F 1554 bolts with supplemental Charpy test requirements shall be specified in applications in which the bolts are subject to seismic loading.

9.2.7   Contract Drawing Representation

High load multi-rotational bearings are generally depicted schematically in the contract drawings. Each bearing manufacturer has unique fabricating methods and procedures that allow it to fabricate a bearing most economically. Depicting the bearings schematically with loads and geometric requirements provides each manufacturer the flexibility to innovatively achieve optimal economy.

9.2.8   Shop Drawing Review

The manufacturer designs and develops shop drawings for high load multi-rotational bearings. The Engineer is responsible for checking and approving the calculations and shop drawings. The calculations shall verify the structural adequacy of all components of the bearing. Each bearing shall be detailed to permit the inspection and replacement of components.

9.2.9   Bearing Replacement Considerations

In some situations, existing bearings, or elements thereof, must be replaced consequent to excessive wear or seismic rehabilitation. Bearing replacement operations generally require lifting of superstructure elements using hydraulic jacks. The designer is responsible for calculating anticipated lifting loads and stipulating these loads on the contract drawings. Limitations on lift height shall also be specified. Consideration shall be given to lift height as it relates to adjacent expansion joints elements and adjoining sections of railing. Stresses induced as a consequence of differential lift height between multiple hydraulic jacks are generally addressed by stipulating restrictions in the plans or special provisions.

Past experience shows that actual lifting loads nearly always exceed calculated lifting loads. Many factors may contribute to this phenomenon, including friction in the hydraulic jack system and underestimation of superstructure dead loads. Unless the Bearing and Expansion Joint Specialist or the Bridge Design Engineer approves a variance, contract documents shall require that all hydraulic jacks be sized for 200 percent of the calculated lifting load. In all cases, the designer shall verify from manufacturer's literature that appropriate hydraulic jacks are available to operate within the space limitations imposed by a particular design situation.

Appendix A
Miscellaneous Design

BRIDGE DESIGN MANUAL
JANUARY 2008

Expansion Joint Details
Compression Seal

COMPRESSION SEAL TABLE

D.S. BROWN		WATSON BOWMAN ACME	
SEAL	WIDTH	SEAL	WIDTH
CV-20	2"	WA-200	2"
CV-2502	2½"	WA-250	2½"
CV-30	3"	WA-300	3"
CV-35	3½"	WA-350	3½"
CV-40	4"	WA-400	4"

TESTING SHALL BE PER AASHTO M-220 PRIOR TO USE.

NOTE:
COMPRESSION SEALS GREATER THAN FOUR INCHES WIDE SHOULD NOT BE USED.

COMPRESSION SEAL
ARMORED OPENING.
USE IN BRIDGE WIDENINGS WITH EXISTING ARMORED JOINTS

- ANGLE SIZE DEPENDS UPON COMPRESSION SEAL USED (TYP.)
- ELASTOMERIC COMPRESSION SEAL
- ⅜" SQUARE BAR

NOTE:
DESIGNER TO USE APPROPRIATE DETAILS FROM THIS SHEET AND CONSULT WITH EXPANSION JOINT SPECIALIST FOR LATEST PLAN SHEET LAYOUT, NOTES, AND UP-TO-DATE DETAILS.

COMPRESSION SEAL
CONCRETE OPENING

- A CONSTR.
- NORMAL TO ℄ OF SEAL
- TEMPERATURE OF STRUCTURE AT TIME OF FORMING EXPANSION JOINT
  - @ 40° F
  - @ 64° F
  - @ 80° F
- FULLY COMPRESSED SEAL HEIGHT
- TOP OF SLAB OR FACE OF CURB
- USE ⅜" EDGER
- ELASTOMERIC COMPRESSION SEAL

[1] USE ¼" FOR ALL SEALS.
[2] USE ½" FOR ALL SEALS.
[3] COMPUTE "A CONSTR." PER EQUATION (12) @ 40°F, 64°F, AND 80°F.
[4] TO BE CHECKED BY THE DESIGNER. SHALL BE LARGE ENOUGH TO PREVENT CLOSURE UNDER THERMAL MOVEMENTS.
[5] SEE BDM SECTION 9.1.3.A. AND DESIGN EXAMPLE FOR COMPRESSION SEAL DESIGN AND SEE "COMPRESSION SEAL TABLE" ON THIS SHEET.

PLAN
EXPANSION JOINT

- SKEW ANGLE < 20°
- SKEW ANGLE > 20°

SEAL CUTTING DETAIL

- DRILL ½"Ø HOLE THRU SEAL. MAKE SURE THE TOP MEMBRANE IS NOT DAMAGED WHEN CUTTING OUT THE WEDGE
- COMPRESSION SEAL
- 0° CORNER "A"
- 45° CORNER "B"
- 2/3 H
- H

SECTION A

- ¼" THICK SYNTHETIC CLOSED CELL EXPANDED RUBBER JOINT FILLER CEMENTED TO JOINT SEAL AT END
- TOP OF ROADWAY
- CORNER "A" SEE "SEAL CUTTING DETAIL"
- CORNER "B" SEE "SEAL CUTTING DETAIL"

Washington State
Department of Transportation

BRIDGE AND STRUCTURES OFFICE

EXPANSION JOINTS

EXPANSION JOINT DETAILS
COMPRESSION SEAL

REGION NO.	STATE	FED. AID PROJ. NO.		SHEET NO.	TOTAL SHEETS
10	WASH.				

JOB NUMBER

M:\STANDARDS\Expansion Joints\COMPRESSION_SEAL_DET.man

Bridge Design Engr.
Supervisor
Designed By
Detailed By
Bridge Projects Engr.
Prelim Plan By
Architect/Specialist

DATE	REVISION	BY	APPR'D

Wed Feb 06 08:04:55 2008

Appendix A
Miscellaneous Design

Expansion Joint Details
Strip Seal

BRIDGE DESIGN MANUAL
APRIL 2008

NOTE:
DESIGNER TO USE APPROPRIATE DETAILS FROM THIS SHEET AND CONSULT WITH EXPANSION JOINT SPECIALIST FOR LATEST PLAN SHEET LAYOUT, NOTES, AND UP-TO-DATE DETAILS.

STRIP SEAL NOTES:

1. DESIGNER SHALL INCLUDE APPROPRIATE DETAILS AND TABLES IN THE PLANS.
2. SEE BDM SECTION 9.1.4B AND DESIGN EXAMPLE FOR STRIP SEAL DESIGN AND FOR DETERMINING OPENING "G" NORMAL TO THE JOINT. FILL IN AMOUNTS CALCULATED AND STEEL SHAPES SELECTED.
3. FOR SKEW ANGLE GREATER THAN 30° SEE JOINT SPECIALIST.
4. SEE JOINT SPECIALIST PRIOR TO SPECIFYING A 5" MOTION RANGE STRIP SEAL.
5. A GROUP 1 PERMITS A ⅜" GAP BETWEEN STEEL SUPPORTING ELEMENTS AT FULL CLOSURE AND A GROUP 2 STRIP SEAL PERMITS FULL CLOSURE.
6. DO NOT USE STEEL SHAPES WITH HORIZONTAL LEGS IN CURB OR BARRIER REGION.

2" MOTION RANGE

GROUP **	MANUFACTURER	ITEM NAME	OPENING "G" NORMAL TO JT. MIN.	MAX.	MIN. INSTALLATION WIDTH NORMAL TO JOINT	@40°F	@64°F	@80°F
	D. S. BROWN	DSB STRIP SEAL A2R-400	½"	4½"	1½"			
2	WATSON BOWMAN ACME	WABO STRIP SEAL SE-300	0"	3"	1½"			
2	R.J. WATSON, INC.	R.J. STRIP SEAL 200	0"	2"	1½"			
1	HEXCEL FYFE CO.	STRIP J200-C ES500	¼"	5½"	1½"			

3" MOTION RANGE

GROUP **	MANUFACTURER	ITEM NAME	OPENING "G" NORMAL TO JT. MIN.	MAX.	MIN. INSTALLATION WIDTH NORMAL TO JOINT	@40°F	@64°F	@80°F
	D. S. BROWN	DSB STRIP SEAL A2R-400	½"	4½"	1½"			
2	WATSON BOWMAN ACME	WABO STRIP SEAL SE-400	0"	3"	1½"			
2	R.J. WATSON, INC.	R.J. STRIP SEAL 300	0"	3"	1½"			
1	HEXCEL FYFE CO.	STRIP J200-C ES500	¼"	5½"	1½"			

4" MOTION RANGE

GROUP **	MANUFACTURER	ITEM NAME	OPENING "G" NORMAL TO JT. MIN.	MAX.	MIN. INSTALLATION WIDTH NORMAL TO JOINT	@40°F	@64°F	@80°F
	D. S. BROWN	DSB STRIP SEAL A2R-400	½"	4½"	1½"			
2	WATSON BOWMAN ACME	WABO STRIP SEAL SE-400	0"	4"	1½"			
2	R.J. WATSON, INC.	R.J. STRIP SEAL 400	0"	4"	1½"			
1	HEXCEL FYFE CO.	STRIP J200-C ES500	¼"	5½"	1½"			

5" MOTION RANGE

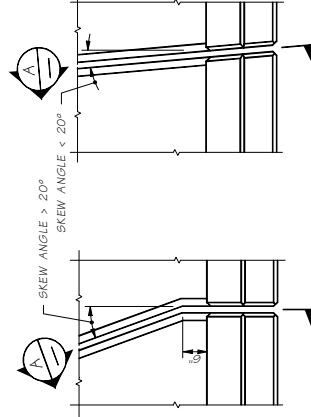| GROUP **	MANUFACTURER	ITEM NAME	OPENING "G" NORMAL TO JT. MIN.	MAX.	MIN. INSTALLATION WIDTH NORMAL TO JOINT	@40°F	@64°F	@80°F
	D. S. BROWN	DSB STRIP SEAL L2-500	½"	5½"	1½"			
2	WATSON BOWMAN ACME	WABO STRIP SEAL SE-500	0"	5"	1½"			
2	R.J. WATSON, INC.	R.J. STRIP SEAL 500	½"	5"	1½"			

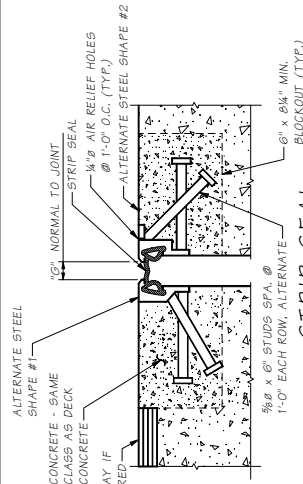* GROUP COLUMN NOT TO BE SHOWN IN CONTRACT PLANS. FOR DESIGN PURPOSES ONLY.

STEEL SHAPE TYPES

MANUFACTURER	ITEM NAME	TYPE	S	T	TYPE	V	W	TYPE	X	Y
D. S. BROWN	DSB STRIP SEAL	SSCM2	1¼"	3¾"	SSA2 *	1¼"	2"	SSGE2 *	1"	1½"
WATSON BOWMAN ACME	WABO STRIP SEAL	M. R. P	1¼"	2¾"	A	1¼"	2"	E	1½"	1½"
R.J. WATSON, INC.	R.J. STRIP SEAL	RJM	3¾"	2¾"	RJA	1¼"	2"	RJE	1¼"	1½"
HEXCEL FYFE CO.	STRIP J200-C	C	2¼"	3"	A	1¼"	2"	E	1¾"	1½"

* TRIM OUTSTANDING LEGS OF SSCM2 SHAPE FOR USE IN TRAFFIC BARRIER

SECTION A — FOR STRIP SEAL / SINGLE SLOPE BARRIER

SECTION A — FOR STRIP SEAL / "F" SHAPE BARRIER

SPLIT STEEL SHAPE AT TRAFFIC BARRIER IF REQUIRED BY MANUFACTURER — 79°, 55°

PLAN ~ EXPANSION JOINT

SKEW ANGLE > 20° / SKEW ANGLE < 20°

STRIP SEAL

- ALTERNATE STEEL SHAPE #1
- ALTERNATE STEEL SHAPE #2
- "G" NORMAL TO JOINT
- STRIP SEAL
- CONCRETE - SAME CLASS AS DECK
- OVERLAY IF REQUIRED
- ¼"ø AIR RELIEF HOLES @ 1'-0" O.C. (TYP.)
- STEEL SHAPE
- ⅝"ø x 6" STUDS SPA. @ 1'-0" EACH ROW, ALTERNATE
- 6" x 8¾" MIN. BLOCKOUT (TYP.)

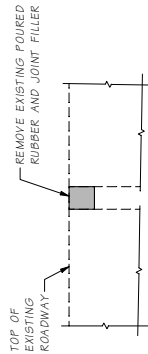STANDARD ANCHORAGE: USE FOR NORMAL TRAFFIC VOLUME. EXTEND SLAB STEEL INTO THE BLOCKOUT
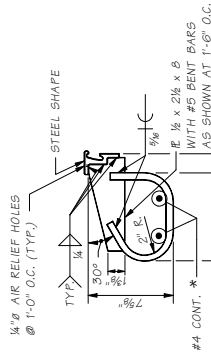
SPECIAL ANCHORAGE: USE FOR HIGH TRAFFIC VOLUME. REQUIRES 9" x 1'-0" MIN. BLOCKOUT. EXTEND SLAB STEEL INTO THE BLOCKOUT. EXTEND BLOCKOUT TO EDGE OF DECK.

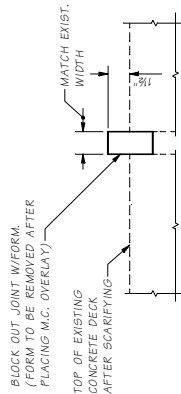* PLACED AFTER EXPANSION JOINT IS IN POSITION. THREAD INTO PLACE FROM THE ENDS.

- #4 CONT. *
- Pl. ½ x 2½ x 8 WITH #5 BENT BARS AS SHOWN AT 1'-6" O.C.
- 30°, 7¾", 2¼", ⅝"R

Bridge Design Engr. / Supervisor / Designed By / Checked By / Detailed By / Bridge Projects Engr. / Prelim. Plan By / Architect/Specialist

REGION STATE — 10 WASH. — FED. AID PROJ. NO. — JOB NUMBER

M:\STANDARDS\Expansion Joints\STRIP SEAL DETAILS.man

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Appendix A

Miscellaneous Design

BRIDGE DESIGN MANUAL

JUNE 2006

Silicone Seal Expansion
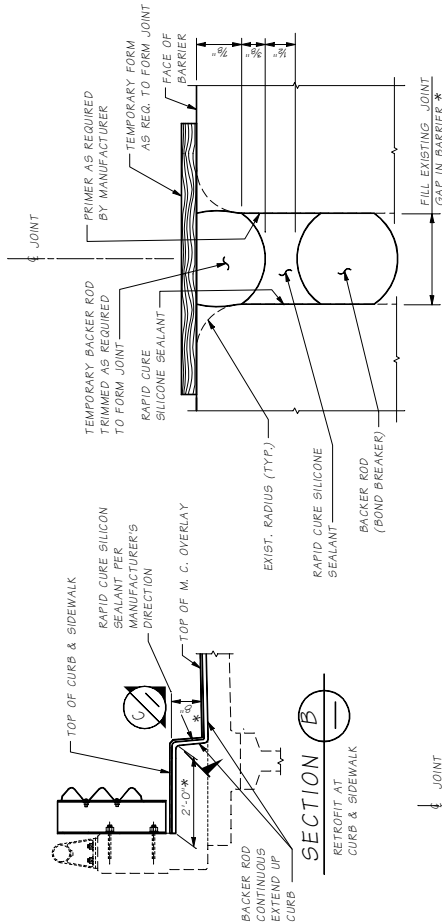Joint Details
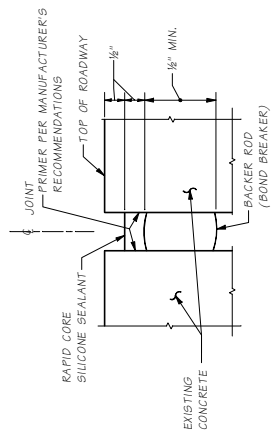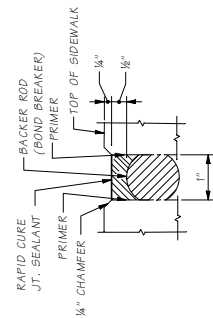
CONSTRUCTION STEPS:

1. REMOVE EXISTING POURED RUBBER & JOINT FILLER MATERIAL FROM EXPANSION JOINT.

2. CLEAN SIDES AND BOTTOM OF JOINT OPENING TO CLEAN AND SOUND CONCRETE.

3. BLOW JOINT OPENING WITH OIL-FREE COMPRESSED AIR TO REMOVE LAITANCE AND DEBRIS FROM REMOVAL OPERATIONS.

4. PLACE FORM IN EXISTING JOINT OPENING TO A HEIGHT LEVEL WITH THE FINAL ROADWAY ELEVATION.

5. PLACE MODIFIED CONCRETE OVERLAY TO FINAL ROW'Y ELEVATION.

6. REMOVE FORM FROM JOINT OPENING AND LIGHTLY SANDBLAST TO REMOVE ALL RESIDUE.

7. PRIME VERTICAL FACES OF JOINT IN ACCORDANCE WITH SEALANT MANUFACTURER'S RECOMMENDATION.

8. PLACE AN APPROPRIATELY SIZED BACKER ROD TO THE CORRECT DEPTH IN JOINT OPENING IN ACCORDANCE WITH SEALANT MANUFACTURER'S DIRECTIONS.

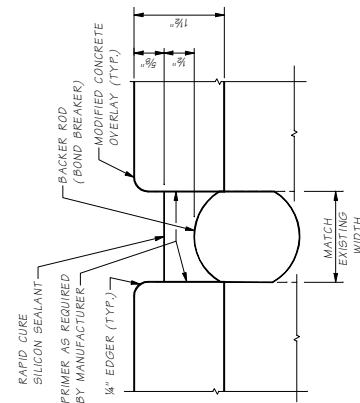9. PLACE RAPID CURE SILICON SEALANT IN ACCORDANCE WITH MANUFACTURER'S DIRECTIONS.

SECTION A
CONSTRUCTION STEP 1
EXISTING EXPANSION JOINT

SECTION A
CONSTRUCTION STEP 4
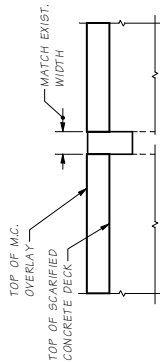
SECTION A
CONSTRUCTION STEP 5

SECTION A
STEPS 6-9

SECTION B
RETROFIT AT
CURB & SIDEWALK

SECTION C
EXPANSION JOINT
MODIFICATION AT
EXISTING BARRIER

SECTION C
AT ROADWAY

SECTION C
AT SIDEWALK

EXPANSION JOINT DETAIL
EXPANSION JOINT MODIFICATION
AT ROADWAY SLAB

REMOVE EXISTING POURED
RUBBER AND JOINT FILLER

TOP OF
EXISTING
ROADWAY

MATCH EXIST.
WIDTH

BLOCK OUT JOINT W/FORM.
(FORM TO BE REMOVED AFTER
PLACING M.C. OVERLAY)

TOP OF EXISTING
CONCRETE DECK
AFTER SCARIFYING

TOP OF SCARIFIED
CONCRETE DECK

TOP OF M.C.
OVERLAY

SEE ROADWAY SLAB JOINT
DETAIL THIS SHEET

RAPID CURE SILICON SEALANT PER
MANUFACTURER'S DIRECTION

TOP OF M. C. OVERLAY

TOP OF CURB & SIDEWALK

BACKER ROD
CONTINUOUS
EXTEND UP
CURB

RAPID CORE
SILICONE SEALANT

PRIMER PER MANUFACTURER'S
RECOMMENDATIONS

TOP OF ROADWAY

JOINT

EXISTING
CONCRETE

BACKER ROD
(BOND BREAKER)

RAPID CURE
JT. SEALANT

PRIMER

TOP OF SIDEWALK

BACKER ROD
(BOND BREAKER)

PRIMER

¼" CHAMFER

JOINT

PRIMER AS REQUIRED
BY MANUFACTURER

TEMPORARY FORM
AS REQ. TO FORM JOINT

FACE OF
BARRIER

TEMPORARY BACKER ROD
TRIMMED AS REQUIRED
TO FORM JOINT

RAPID CURE
SILICONE SEALANT

EXIST. RADIUS (TYP.)

RAPID CURE SILICONE
SEALANT

BACKER ROD
(BOND BREAKER)

FULL EXISTING JOINT
GAP IN BARRIER *

BACKER ROD
(BOND BREAKER)

MODIFIED CONCRETE
OVERLAY (TYP.)

RAPID CURE
SILICON SEALANT

PRIMER AS REQUIRED
BY MANUFACTURER

¼" EDGER (TYP.)

MATCH
EXISTING
WIDTH

EXPANSION JOINTS

Washington State
Department of Transportation

BRIDGE
AND
STRUCTURES
OFFICE

SILICONE SEAL EXPANSION
JOINT DETAILS